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AMERICAN SOCIETY OF CIVIL ENGINEERS

BASIC REQUIREMENTS FOR MANUSCRIPTS

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. Authors should indicate the technical division to which the paper is referred. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles must have a length not exceeding 50 characters and spaces.

2. A summary of approximately 50 words must accompany the paper, a 300-word synopsis must precede it, and a set of conclusions must end it.

3. The manuscript (an original ribbon copy and two duplicate copies) should be double-spaced on one side of 8½-inch by 11-inch paper. Three copies of all illustrations, tables, etc., must be included.

4. The author's full name, Society membership grade, and footnote reference stating present employment must appear on the first page of the paper.

5. Mathematics are recomposed from the copy that is submitted. Because of this, it is necessary that letters be drawn carefully, and that special symbols be properly identified. The letter symbols used should be defined where they first appear, in the illustrations or in the text, and arranged alphabetically in an Appendix.

6. Tables should be typed (an original ribbon copy and two duplicate copies) on one side of 8½-inch by 11-inch paper. Specific illustrations and explanation must be made in the text for each table.

7. Illustrations must be drawn in black ink on one side of 8½-inch by 11-inch paper. Because illustrations will be reproduced with a width of between 3-inches and 4½-inches, the lettering must be large enough to be legible at this width. Photographs should be submitted as glossy prints. Explanations and descriptions must be made within the text for each illustration.

8. The desirable average length of a paper is about 12,000 words and the absolute maximum is 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

9. Technical papers intended for publication must be written in the third person.

10. The author should distinguish between a list of "Reading References" and a "Bibliography," which would encompass the subject of his paper.

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Vol. 87 No. HW 2

Journal of the
HIGHWAY DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

June, 1961

Papers

Page

DISCUSSION

(over)

Note.—Part 2 of this Journal is the 1961-20 Newsletter of the Highway Division.

The three preceding issues of this Journal are dated September 1960, December 1960, and March 1961.

Journal of the
HIGHWAY DIVISION
Proceedings of the American Society of Civil Engineers

INTEGRATED PARKWAY AND EXPRESSWAY PLANNING[a]

By D. Jackson Faustman,[1] F. ASCE

SYNOPSIS

For a balanced transportation system there must be an integration of expressways, local streets and highways, public transit, and parking facilities. Future parking requirements, particularly in central districts, are sizeable and should be recognized in the development of the city master plan. Broad design concepts are outlined for locating parking in relation to the highway network and the land use patterns. Analysis is made of the specific problems of access between expressways and parking facilities. The possibilities of parking under or over highways are explored and the experiences of various cities in this regard are outlined.

The amount of parking to be provided must be increased over current levels. This will involve new financing policies and methods possibly involving more extensive use of funds from parking meters. There is need for better data on parking requirements, and considerable work is required to improve the functional design terminal of facilities.

THE PROBLEM

The most prodigious roadbuilding program in history is under way (in 1961). Motor vehicle movement into and through urban areas will be greatly improved as a result of construction financed with Federal and State funds. The broad

Note.—Discussion open until November 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Highway Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. HW 2, June, 1961.

[a] Presented at the October 13, 1960 ASCE Annual Meeting at Boston, Mass.

[1] Consulting Traffic Engr., Sacramento, Calif.

objective of this program is the development of a total transportation system in which all components are in reasonable balance. However, examination of the results achieved, particularly in the urban areas, reveals that there is a serious unbalance in that facilities for parking are lagging far behind the facilities for movement. Greater effort and resources must be applied to the parking problem in order to bring parking facilities within reasonable balance with the movement facilities planned for or under construction.

The reasons for the parking dilemma are many and complex. Parking has historically been considered a local problem. At the local level there has been a critical lack of planning for parking. In many instances the provision of parking (except at the curb) has been left entirely up to private enterprise. It is only recently that off-street parking has come to be regarded as a legitimate, expected, and necessary public municipal service. The American Municipal Association (AMA) policy statement on parking states, "For the welfare, safety, and convenience of its citizens, and to insure its own future existence, each municipality must accept the responsibility for definite action which will result in the furnishing of adequate, permanent parking facilities."

Whereas municipal government in some cases has provided expressways (the words "freeway" and "expressway" are used interchangeably herein), it is the state and federal government for the most part who finance and construct this type of modern highway facility. These agencies generally confine their operations to "highway purposes." Thus, parking has not been given the attention it properly deserves. Physically and functionally, parking and expressways are intimately related and integrated planning of both facilities is essential to sound transportation improvement and orderly community development.

S. S. Taylor, General Manager of the Department of Traffic for the City of Los Angeles has indicated the problem in stating that, "A vehicle that cannot be left and picked up again is virtually useless. In the last analysis the automobile has only one destination—a parking space . . . The traffic capacity of the freeway or local street, . . . is actually dependent upon the capacity and accessibility of parking spaces in the metropolitan terminal areas. . . . to say that the off-street parking problem is a local concern is to avoid the issue." It is the purpose of this paper to explore various facets of this problem.

PARKING DEMAND

How serious is the parking problem? As with many urban problems it is relative. In the larger metropolitan areas the problem is extremely critical when measured in terms of inability to park. In many smaller communities no difficulty is experienced in finding a parking space, although the space may not be as close to the parker's destination as he would wish. (We Americans are forgetting how to walk!)

Parking demand is concentrated in the core area of the city. Because most expressways are on the perimeter of the core, the parking demand is usually some distance from the highway facility. Demand is made up to two types of parkers, the short-time parker on a shopping or business trip and the long-time (all-day) parker working in the area. The short-time parker will not walk far to his destination (the smaller the city, the shorter the distance) whereas the long-time parker walks some distance. This is because the long-time parking facilities with the lower rate structures are generally on the fringe

of the central core. It follows then that the parking demand near the perimeter expressways will be for long-time parking.

FUTURE PARKING REQUIREMENTS

What is the magnitude of the future parking needs? Two specific examples are offered as indicative of the size of the problem. The Washington, D. C., transportation study contains some interesting statistics. If an auto-dominant system of highways were developed, the parking requirements in downtown Washington are estimated in 1980 to be 85,000 new off-street spaces. This is equivalent to about forty garages as large as the Chicago, Illinois Grant Park underground garage. The total cost is estimated at approximately $236 million, even assuming that the great demand for appropriate sites for garages would not inflate land values. The consultants concluded that there were serious engineering questions as to the practicality of an auto-dominant plan and made appropriate recommendations for the use of rapid transit, that, of course, minimizes parking needs.

The needs of smaller cities are similarily startling. Studies in Sacramento, Calif., indicate that some 21,500 new parking spaces will be needed by 1980 to serve the anticipated increased motor vehicles coming into the downtown area on a proposed freeway network. If all these vehicles were to be accommodated in surface parking facilities, it would require the clearing of approximately one-third of the entire land area in the central business district, even allowing for continued curb parking.

THE CITY OF TOMORROW

These parking requirements are possibly fantastic and unrealistic. The difficulties that are being experienced throughout the country with the development and financing of rapid transit, brought about by the reluctance of the American motorist to give up his car, should serve as a warning that these tremendous parking demands may have to be faced. That this is not too far off is illustrated in Fig. 1 that shows the extent to which the auto has taken over downtown Los Angeles, Calif. Darkened areas are those occupied by or for the use of autos. E. M. Khoury F. ASCE, proposed a super-plan that would remake the city for the automobile. Noting that the freeways empty their heavy loads into narrow downtown streets never built for such traffic, Khoury proposes that freeways be knifed through the downtown area as elevated highways at selected intervals over the present street grid and that these new elevated freeways be fed directly into combination multi-level garage and office structures built around them. The effect would be to create a second independent traffic system over the existing one and if necessary the third and fourth above that. Ultimately the street would be left entirely to the pedestrian. The plan at least has the virtue of carrying things to their ultimate conclusion—remaking the city entirely in the image of the automobile. No one knows how much such a plan would cost, but by any computation the additional expense of garaging a city's entire traffic would be staggering.

There is a wide range of predictions on the future impact of the auto on the urban area. There are those who see absolute chaos resulting from rapid motorization and urbanization. It was generally noted at the International Congress

of Geographers in Stockholm, Sweden in August 1960 that the central core of large cities would disintegrate in spite of all current efforts to preserve it because of its inability to accommodate the auto.

These dire warnings of calamity have been summarized:[2]

> Unless some way is found to divert in part the motorist from his controlled desire to plunge his space-consuming vehicle into the maelstrom of the center city, these problems will mount and the tiered, clamorous, more-than-slightly-monstrous, Babylonian auto-city is not far off.

At the other extreme, there appear to be, unfortunately, many responsible officials who are inclined to shrug off the problem of the auto. It is encouraging to find increased attention being given to coordinating highways and urban development through such devices as joint committees of the AMA, the Association of State Highway Officials (AASHO), the Highway Research Board (HRB),

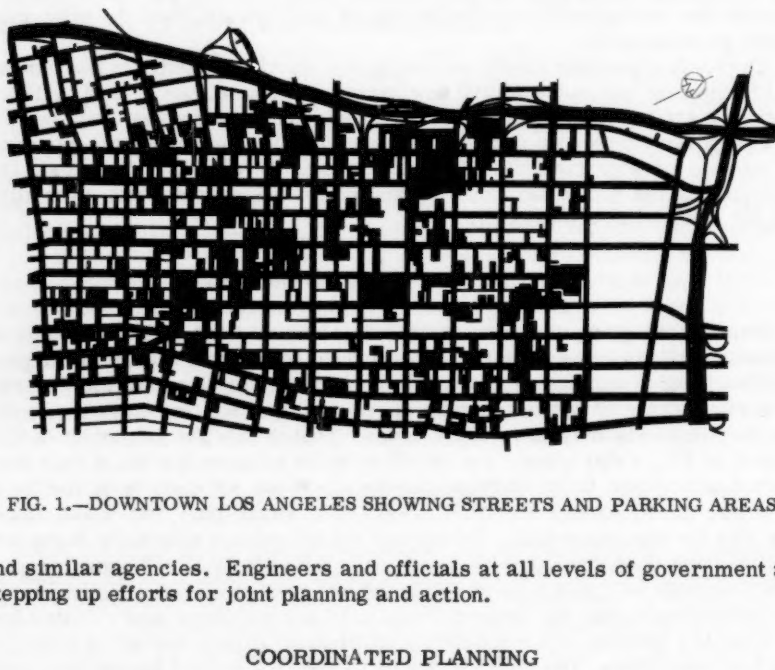

FIG. 1.—DOWNTOWN LOS ANGELES SHOWING STREETS AND PARKING AREAS

and similar agencies. Engineers and officials at all levels of government are stepping up efforts for joint planning and action.

COORDINATED PLANNING

The basic need in coordinating the planning of parking and expressways is leadership to bring together the factions involved, highway engineers, city planners, parking operators, municipal officials, and so on. The mayor or city manager can set the machinery in motion and there is no question that the citizens will support the effort enthusiastically.

Special note should be taken of the efforts of the Transportation Institute of the University of Michigan, Ann Arbor, Mich. In 1959 the Institute, under the

2 Architectural Forum, Time Inc., New York.

direction of John C. Kohl, M. ASCE, and with the cooperation of the Automotive Safety Foundation (ASF), brought together various experts in a conference on coordination of expressways and parking.

The single, most important, basic tool for accomplishing the needed integration is the city's master plan. This plan is an all-inclusive, comprehensive, long-range pattern for the future development, or redevelopment of city. It is not a single plan, but a series of coordinated plans that aims to encourage the best possible use of all the land within the city's boundaries and to establish the general location of all the contemplated public improvements. Parking should be considered as a public improvement and a long-range plan to be developed with specific sites, wherever possible, properly coordinated with the street and highway elements of the master plan.

It is essential that the parking elements of the plan be developed at the same time as the street and highway plan. This practice has not usually been followed. It is not proper to have independent groups planning the street and highway system of the city, the planners analyzing the land use and zoning, and still another agency such as a parking authority developing terminal facilities. All elements of the plan must be integrated as part of a team function. Unless parking is related to the freeway at the design stage, opportunities for subsequent physical correlation will be limited.

The writer was impressed with the coordinated urban transport planning foun in Scandinavia. In Stockholm, for example, the master plan[3] includes location of expressways, all types of mass transit (underground, interurban railways, street cars, and buses) and parking facilities. This report and the opinion of Sven Lundborg, Chief Research Officer, Department for City Planning, indicate that in the central area the location of parking facilities is governed by the demand for short-term parking from business enterprises, insofar as this can be satisfied without detriment to the capacity for moving traffic. Park and ride facilities are featured at the approaches to the inner city.

The city planners in Stockholm realize that not only must the different modes of transport and parking be coordinated with each other, but also that planning of other phases of city development must be done in close cooperation with the transport planners.

PLANNING CONCEPTS

A broad planning concept for integrating parking into the total plan visualizes a central core of buildings (that may be served by relatively narrow streets or no vehicular trafficways whatsoever) surrounded by parking fields or multideck structures. Through traffic would bypass the central city on ring freeways whereas workers and shoppers from all directions would use these same freeways to reach the parking areas nearest their destination. Under urban renewal (redevelopment) a number of bold schemes are now being examined that incorporate these general concepts. Urban renewal offers the engineer an excellent chance to bring about better integration of parking and freeways. Blighted areas near the central core are good locations for the perimeter freeways and land is usually available for the needed parking. However, this opportunity is

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3 Comprehensive Traffic Plan, Master Planning Commission of Stockholm, Sweden, February, 1960.

not being exploited to the fullest for the following reasons:

1. The freeway builders are under pressure to get the freeways in operation;
2. Urban renewal is a new governmental process that takes time to generate the public support it needs to advance at the same rate as the freeways.

Notable examples of this type of planning include the Philadelphia Center City Plan, (Philadelphis, Pa.) to rejuvenate the downtown core. This plan calls for eight parking facilities at the edge of the central area with ramp access to an expressway loop. The Golden Triangle redevelopment in Pittsburgh, Pa., the Hartford, Conn. downtown renewal program, and the New Haven, Conn. central district improvement plan all follow the concept of locating major off-street parking facilities conveniently between the heart of the business district and expressway loops. Fig. 2 shows elements of the New Haven plan.

Perhaps the best known and most dramatic scheme that has been advanced for carrying out this planning concept is Victor Gruen's Fort Worth, Texas Plan. In this scheme there is a perimeter freeway around the core area. Extending into the core would be six off-street parking garages, equipped with escalators and moving sidewalks. Each one of these garages would accommodate 10,000 vehicles and would be connected directly to this perimeter freeway. No difficulties in handling the rush hour movements were anticipated on the basis of experience with a number of 10,000 car shopping centers currently in operation. No action has been taken (as of 1960) on Gruen's plan because the most extensive redevelopment plans usually require many years before their realization.

EXPRESSWAY DESIGN

The total design of the highway facility (alinement, profile, cross-section, and so on) is not significantly affected by parking volumes per se. Traffic volumes to and from parking facilities are not large enough to influence interchange design. In some cases the ramp patterns and locations may have to be considered in light of peak hour surges to and from major parking facility. Additional ramps may be required to accommodate traffic from single large traffic generators such as a stadium, ball park, or convention hall. Movements to and from parking facilities must not interfere with expressway traffic movement.

LOCAL STREET CONNECTIONS

Parking should be readily accessible to streets connected with the expressway. The facilities should not be too close to impair off-ramp movement but close enough that traffic does not have to traverse the city streets any more than absolutely necessary. In effect, the connecting street becomes a "deceleration lane" for the parking facility. The capacity of street connections is often critical and it is usually helpful to make the streets one-way. This will avoid the left turn conflicts that occur at parking facility entrances and at intersections. Ramp connections are often designed to fit into one-way street patterns.

A basic decision is required as to whether the parking should be accessible from both directions as related to the freeway. This factor dictates such items as number and location of parking facility access points, direction of one-way streets and their freeway connections.

Detroit, Mich. has planned its municipal parking so as to use local streets as connectors to the nearby expressways. One such facility is an all-day, 600 car, 3-level, ramp-type garage located two and one-half blocks from an expressway. The motorist can park his car within a minute after he leaves the expressway and when leaving for home he is on the expressway a minute after leaving the garage. This, obviously, has a distinct appeal for motorists. A survey

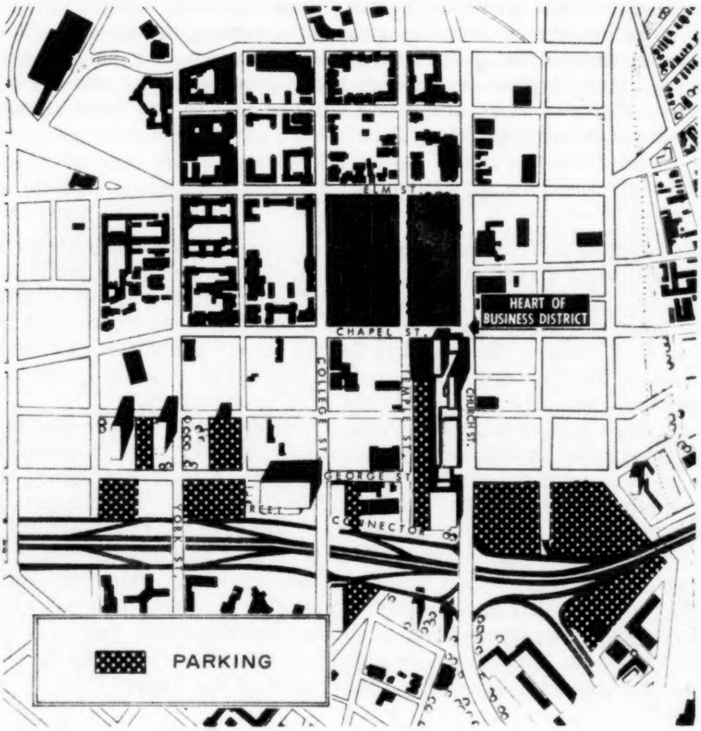

FIG. 2.—NEW HAVEN, CONN. DOWNTOWN IMPROVEMENT PLAN

shows that about half the parkers at this facility walk more than 1200 ft to reach their destination. Similarly, five other large parking facilities of the Detroit Parking Authority have convenient access to the expressways. The downtown streets will be relieved of the burden of carrying an estimated 1-1/2 million cars annually through the appropriate location of these major off-street park-

ing facilities. In Detroit a liaison is maintained with the Coordinator of Expressways and the Parking Authority to insure that this program will be continued.

PARKING FACILITY OPERATION

The operating methods of the parking facilities are important because they should be able to receive entering cars at a rate rapid enough to prevent backups into the street system or expressway, or both. Self-parking facilities and the use of devices for automatic space control and assignment make for more rapid entrance rates. Particular attention should be paid to the design of the facility entrance and reservoir capacity. Most parking facilities have entrances at right angles to the street or the main traffic stream. Such entrances tend to slow down the street travel because of the turning maneuvers. There are notable exceptions to this type of right angle entrance design, such as the Grant Park Garage in Chicago wherein the main entrances and exits are in the center of Michigan Avenue. More attention should be paid to this point in the design of future large off-street parking facilities.

DIRECT RAMP CONNECTIONS

The difficulties in a central district of providing closely-spaced interchanges probably will limit direct ramp connections. Such connections would be justified, however, where there are major parking structures accommodating extremely large volumes. There are only a few instances where such connections have been made or planned. The Detroit convention hall has a direct ramp connection. In Hartford the new east-west expressway runs adjacent to a 1000 car garage and has direct access to the parking structure.

For direct-type connections to function successfully it is essential that adequate parking capacity with smooth operating characteristics be present in the facility so as to not block expressway movements. A means of closing the connecting ramps when the facility is full is a desirable feature. Appropriate signing and signal systems will be necessary to advise in advance when the garage is full. The previously mentioned Detroit Garage has this feature on the expressway. Aside from design considerations the direct-ramp connection to a privately owned facility may require legislative approval because, generally, the state highway department will construct roadways only into or onto public right of ways. An interesting problem also arises in regard to the financing of connecting ramps: Are these ramps part of the highway or the parking structure?

The direct-ramp connection to major parking facilities is not confined to large parking structures. There are a number of regional shopping centers and industrial plants that provide this type of terminal service. The connections, however, are usually made through the ramps to the local streets, that, in turn, feed the parking areas. A well known design of this type is the Pentagon in Washington, D. C., where the various parking areas are all connected directly to the perimeter freeway system.

PARKING UNDER OR OVER EXPRESSWAYS

Although possibly limited in application, mention should be made of the desirability of using areas under or over expressways for parking. There are

several benefits which would accrue where this joint use of right of way is practical, such as; (1) economy of site acquisition; (2) conservation of tax base, that is, minimizing the withdrawal of taxpaying land for public uses; and (3) acceleration of the development of abutting lands by providing parking.

In California it is the policy of the State Division of Highways to permit the use of space under freeway structures for parking when this does not interfere with the prime purpose for which the structure was built—the moving of traffic. Section 112 of the Federal Highways Act of 1956 limits use of right of way on any interstate system for parking purposes only. The philosophy that governs these policy decisions is apparently similar to that which applies to the connections of freeways or highways to city streets. Incidental improvements at the point of entrance and exit to the streets may be included in highway construction, but the general principle followed by the state is that the provision for street travel is an obligation of the local government. Provision of parking facilities other than those that can be provided incidental to the highway facilities is considered by the State Highway Department to be the obligation of local government.

In carrying out this policy the California Division of Highways routinely paves and fences the area under elevated freeways. These areas are leased to successful bidders at public auction with the term of the lease not exceeding 10 yr with 5 yr being the usual term. The policy of the U. S. Bureau of Public Roads, Dept. of Commerce (BPR) for parking on portions of the interstate highway system differs from that of California in that the Bureau wants the state highway department or a state parking authority to operate the parking facilities.

Extensive areas are available and in use under the elevated freeways in the San Francisco, Calif. area. There are no opportunities in the Los Angeles freeway system for the development of any parking areas. In San Francisco, under the James Lick Skyway there are 2,445 parking spaces available in the 685,000 sq ft beneath the structure. Of the total available, 1,435 spaces have been leased. Under a portion of the Embarcadero Freeway, 988 spaces are available in 276,500 sq ft. Eight hundred and eighty of these have been leased. Beneath the Thirteenth Street extension there is a total space of 239,000 sq ft, with 855 parking spaces, of which 474 have been leased to date (October, 1960). Under the Bay Bridge approaches, total available area is 554,000 sq ft, that will permit 1437 spaces for parking, of which 1,360 are leased.

The total of these figures is 5,745 available spaces, of which 4,149 are leased (as of October, 1960). In the future there will be 1,500 additional spaces available in connection with the construction now under way on the Embarcadero and Central Freeways. There will thus be 7,245 spaces available in the city for this type of parking. In addition to this, 383,000 sq ft of truck and bus parking is in use.

In Oakland, Calif., 1,345 spaces are available, of which 584 have been rented. More will be made available as construction proceeds. The rental varies between 0.67 and 2.7 cents per sq ft per month. Parking charges vary from 25 cents to 65 cents per space per day—monthly charges from $5 to $12.50 per space. These freeways are on the fringes of the central core and the parking provided is patronized by all-day parkers working in the area or willing to walk the relatively long distance to their place of employment.

Other cities in which parking under expressways is practiced include Orlando, Fla., New Orleans, La., and Boston, Mass. New York, N. Y. proposes to establish parking under the elevated structure of the Brooklyn-Queens Express-

way in the downtown area and under the Bruckner Expressway near the Bronx business district.

There has been no extensive use of the space above expressways for parking structures. This type of facility would be particularly applicable for expressways through topography requiring extensive cut sections. Several years ago such a plan was considered in Seattle, Wash. where sections of the Tacoma-Everett Freeway passed near the central district. No action has been taken on this scheme (as of October, 1960). Detroit considered the possibilities of using air space but dropped the matter because of excessive cost in relation to location. The design of a garage structure over an expressway presents some difficult problems in framing and foundations because of long spans required to fit the expressway cross-section.

It is not uncommon to find parking lots in leftover slices of expressway right of way developed either privately or by state or local government. Examples of this practice are to be found in Boston along the John F. Fitzgerald Expressway.

CONCLUSIONS

It is essential that better ways be found to integrate freeways with existing streets, parking facilities, public transit, and the patterns of urban land use. The first step involves an effective system of cooperation between those responsible for freeway design and construction and the local agency charged with terminal facility development.

The amount of parking to be provided must be increased over current levels. This will involve new financing policies and methods possibly involving more extensive use of funds from parking meters.

The optimum relationship between expressway and parking facility location and access involves the use of local streets with expressway ramp connections. Direct connection of an expressway to a parking facility poses difficult design and traffic control problems. Parking must not interfere with the prime purpose of the expressway—the moving of traffic.

Problem areas where much needs to be done include better data on parking needs, financing methods, legislative authority, and terminal facility design.

The end objective is a transportation system wherein a balance is achieved between the facilities for the motor vehicle at rest as well as in motion, the mass transit of people and goods, and for the interchange of one form of transportation to another.

Journal of the
HIGHWAY DIVISION
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DISCUSSION

ROADBEDS ON HIGHWAYS AND AIRPORT RUNWAYS[a]

Closure by Ira B. Mullis

IRA B. MULLIS.[10]—The concept on which this research is based is that roadbeds can best be designed by the well-known rational method applied to the design of bridges and pavements. Moreover, as roadbeds must resist forces such as loads and weather, they must be rigid bodies; therefore, air and water must be restricted to very small percentages in the roadbeds.

Roadbed Design Based on Stress-Strain Method.—The discussion by Jacob Feld, F. ASCE plainly indicates that he recognizes the need for a rational method in the design of roadbeds, similar to that followed in the design of concrete mixes and steel bridges. The evidence of his hopefulness seems apparent when he mentions terms such as physical stresses and strains, extrapolation and other terms well-known to the engineer.

Concerning Feld's apparent hopefulness, it seems certain that a method for designing a roadbed to sustain certain loads and resist weather has been found. Although much effort has been made to find a condition in which the method fails to apply, no such condition has been found, and evidence of its value grows stronger as the research continues.

Tables 9, 10, and 11 are based on strength measured by two different methods. Results of the triaxial tests made by Gerald A. Leonards, M. ASCE (34)[11] are shown in Table 20 and Fig. 4. The magnitude of the strength is arranged in decreasing order from the top to the bottom of the table. Density values, expressed in pounds per cubic foot in the original table, have been converted into grams per cubic centimeter to facilitate computations. Although the effects of pore space and saturation on compressive strength have been shown for Mississippi loess and Ft. Union clay, only that of the Mississippi loess is indicated. Each indicates similar trends; hence, only the Mississippi loess table is shown, though both loess and clay are shown in Fig. 4.

Maximum, minimum, and average values are shown for the Mississippi loess in Table 20. As total pore space and saturation do not have as wide range as oftentimes found in many tests, the table is well-adapted to the demonstration of the effects of total pore space and its saturation on compressive strength.

For plotting the line of average relationship of pore space when 90.3% saturated, saturation values between 88% and 92% only are included, but other values are plotted along the line with saturation values given to the nearest whole unit. This method of plotting is followed to give the most precise re-

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a March 1960, by Ira B. Mullis (Proc. Paper 2412).

10 Cons. Engr. (Retired), Raleigh, N. C.

11 Numerals in parenthesis refer to corresponding items in the Bibliography.

lationship between strength and pore space when 90% saturated. Saturation values much greater or less than 90% are given in the graph to the nearest

TABLE 20.—TRIAXIAL COMPRESSION TESTS ON MISSISSIPPI LOESS[a]

Series and No.	Density, in grams per cubic centimeter		Weight percent water		Pore space		Compressive strength in kilograms per square centimeter
	Apparent	Bulk	Capacity	Content	Total percentage	Saturation percentage	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
3- 9	2.72	1.77	19.3	17.7	34.8-	89.8	10.49+
3- 7	2.72	1.77	19.3	17.9	34.8	91.7	8.94
3- 3	2.72	1.75	20.4	17.8	35.6	87.2	7.05
1- 8	2.72	1.76	20.0	18.0	35.3	97.9+	6.90
2- 7	2.72	1.74	20.7	17.9	36.0	86.4	6.02
5- 9	2.72	1.72	21.3	19.9	36.7	93.4	5.55
3- 4	2.72	1.74	20.7	17.9	36.0	86.5	5.54
1- 7	2.72	1.72	21.3	18.1	36.7	89.1	5.46
3- 6	2.72	1.73	21.0	18.0	36.3	85.6	5.39
2- 5	2.72	1.74	20.7	17.8	36.0	86.0	5.24
3-10	2.72	1.72	21.3	19.1	36.7	89.6	4.68
3- 5	2.72	1.72	21.3	18.1	36.7	85.0	4.40
2- 4	2.72	1.68	22.8	17.5	38.2	76.7	4.20
5-10	2.72	1.68	22.8	21.0	38.2	92.0	3.72
5-14	2.72	1.68	22.8	20.9	38.2	91.6	3.62
1- 6	2.72	1.63	24.5	18.4	40.0	75.0	3.60
2- 3	2.72	1.62	24.9	17.5	40.4	70.2	3.05
5-15	2.72	1.66	23.5	21.0	39.0	89.2	2.98
5- 5	2.72	1.66	23.5	21.5	39.0	91.4	2.89
5- 7	2.72	1.64	24.2	21.9	39.7	90.5	2.32
2- 6	2.72	1.59	26.1	18.1	41.5	69.2	2.26
5-13	2.72	1.62	24.9	22.7	40.4	91.0	1.91
5-11	2.72	1.62	24.9	22.5	40.4	90.2	1.82
5- 6	2.72	1.61	25.4	22.8	40.8	89.7	1.62
1- 6	2.72	1.45	32.2	18.5	46.6+	57.4-	1.43
5- 3	2.72	1.59	26.1	23.4	41.5	89.6	1.32-
Average,		1.68	22.8	19.4	38.2	86.0	4.30

Maximum and minimum strength with its influencing values are shown by the signs + and - respectively.

[a] Basic data, Cols. 1, 2, 3, 5, and 8 are from "Strength Characteristics of Compacted Clays," by Gerald A. Leonards, Transactions, ASCE, Vol. 120, 1955, Table 3. Maximum dry unit weight (Standard Proctor) 108.4 (1.74 g per cu cm). Maximum dry unit weight (Mod. AASHO) 120.7 pcf (1.93 g per cu cm). Optimum moisture (Standard Proctor) 18.0%. Liquid limit 37%. Optimum moisture (Mod. AASHO) 12.8%. Plastic limit 23%. Bulk density originally expressed in pounds per cubic foot, is here expressed in grams per cubic centimeter to facilitate computations. 1 kg per sq cm is equivalent to 14.2 psi. + = Maximum; - = minimum.

whole number. Obviously, distance of points from the average 90.3% line indicates the precision of the tests.

A close examination of Fig. 4 shows that information may be obtained as follows:

1. Additional values of usable precision may be interpolated or extrapolated from the lower limit of proportionality almost to zero pore space.

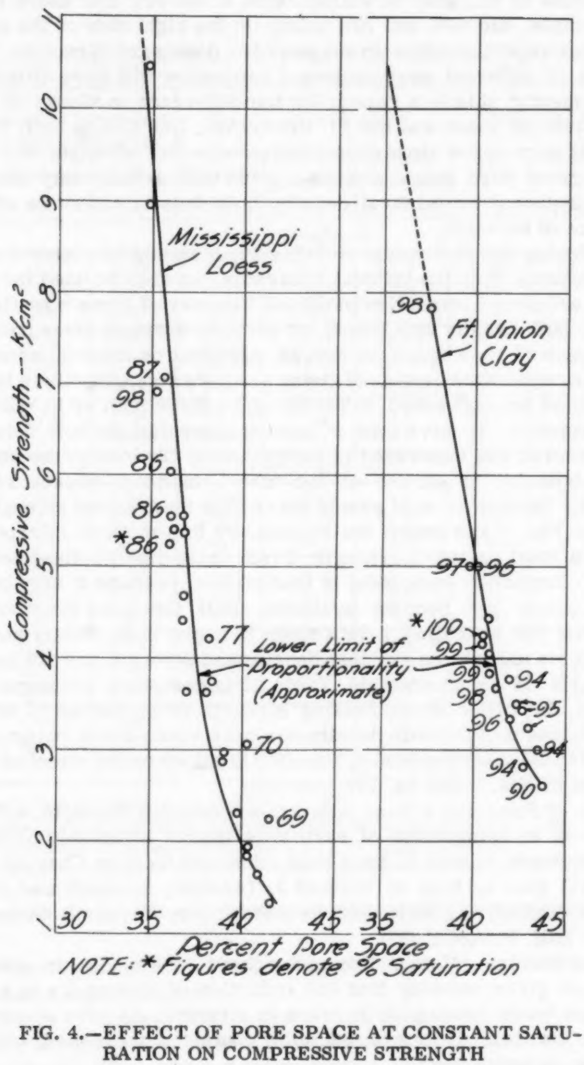

FIG. 4.—EFFECT OF PORE SPACE AT CONSTANT SATURATION ON COMPRESSIVE STRENGTH

2. By means of the graph, one may determine the lower limit of proportionality that may well be considered as the upper boundary region of excessive weakness.

3. In case the indicated strength of a specimen is based on a percent of saturation not located on the 90% line of the graph, higher or lower strengths may be determined by extrapolating from two or more points of identical saturation not on the 90% line.

For example, any of the three points on the Mississippi loess graph with a saturation value of 86% may be extrapolated to the 90% line above it. Also, as another example, the 69% and 70% points on the right side of the same graph also may be extrapolated either in an upward or downward direction. Obviously, pore spaces of different percentages of saturation will have different slopes also; consequently, this is a reason for the difference in slopes of the graphs of the Mississippi loess and the Ft. Union clay. Due to this fact, it should be apparent that pore space alone cannot indicate the true strength of a specimen. Also, as a given pore space attends a given bulk density only when attended by a given apparent or mineral density, bulk density cannot be utilized as a true measure of strength.

Before closing the discussion of Table 20, it should be stated that less expensive apparatus than the triaxial apparatus can also be used for this work. Indeed, any accurate compaction mold and hammer of types now (1961) in use may provide bulk density specimens for suitable strength measurements. For the computation of pore space, of course, apparent or mineral density is also essential. Any apparatus capable of giving accurate strength values is adequate. Strength should be expressed in pounds per square inch or in kilograms per square centimeter. To save time in making computations bulk density should also be measured and expressed in metric units. Obviously, the strength of a roadbed should be in excess of the wheel load it is expected to support; therefore, the laboratory man should determine the greatest strength possible as shown in Fig. 4 and report the results. By this method, interpolation will disclose additional values of strength. From these results, final decision may be made. If compaction equipment is unavailable, perhaps it may be obtained. It will certainly not become available until the need for it is indicated.

Concerning the so-called relationship between bulk density and strength, Campen and his associates say that density and strength are not proportional and cite Table 18 as an example. Table 21 is compiled, recomputed and rearranged in the order of decreasing strength from Tables 17 and 18. This table shows that neither bulk density nor pore space alone governs strength. With regard to the pavement bases, attention is called to the effect of difference in saturation in Nos. 9 and 9a, 11a, and 11b.

Reduction of Pore Space Near Saturation Increases Strength.—The concept that increases in compaction of earthwork do not invariably produce an increase in strength seems to have been advanced first by Charles R. Foster, F. ASCE (35) then in turn by Willard J. Turnbull, F. ASCE and Foster (36), but later the concept was declared to be untenable by Harold J. Gibbs, M. ASCE and Jack W. Hilf, F. ASCE (37).

Under the heading, "Compaction of Roadbeds," the concept is again rejected with citations given showing that the reduction of pore space in a given material always tends toward an increase in strength. As pore space in a given material is lowered, of course, the solid content is increased; consequently, strength also is increased.

In an effort to show that reduction in soil strength does not follow increases in bulk density, data from Turnbull and Foster's Fig. 2 (36), entitled, "The

Relationship Between CBR and Density for Various Laboratory Water Contents" are expressed in Table 22, Columns 1, 2, 4, and 8 and the remaining values, Columns 3, 5, 6, and 7 are computed therefrom.

To facilitate the comparison of values in Table 22, the data are arranged in the order of increasing molding water content and grouped according to pore space. As all specimens are made from the same sample, of course, the

TABLE 21.—INTERRELATIONSHIP OF PORE SPACE, SATURATION AND STRENGTH[a]

Specimen No.	Density grams per cubic centimeter		Weight percent water			Pore space		Strength, in psi
	Apparent	Bulk	Capacity	Content (original)	Content (future)[b]	Total	Saturation	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
(a) Roadbeds								
10	2.65	1.99	12.4	10.6	11.2	24.8	85.4	1,362
10a	2.65	1.88	15.4	13.2	13.9	29.0	85.7	353
10b	2.65	1.82	17.2	15.0	15.5	31.3	87.1	258
13	2.60	1.75	18.7	16.0	16.8	32.6	85.5	828
13a	2.60	1.71	19.9	17.9	17.9	34.1	89.9	501
13b	2.60	1.62	23.3	18.5	21.0	37.7	79.4	278
2	2.7	1.86	16.7	15.7	15.0	31.0	94.0	605
2a	2.7	1.80	18.5	17.0	16.6	33.3	91.9	376
(b) Pavement Bases								
7	2.63	2.30	5.4	3.7	4.6	12.5	68.5	2,419
7a	2.63	2.26	6.2	5.0	5.3	14.0	80.5	834
7b	2.63	2.25	6.4	5.8	5.4	14.4	90.6	137
9	2.61	2.28	5.5	4.4	4.7	12.5	80.0	1,974
9a	2.61	2.28	5.5	4.8	4.7	12.5	87.2	1,416
9b	2.61	2.27	5.7	4.8	4.8	13.0	84.0	1,273
11	2.64	2.16	8.4	6.9	7.1	18.1	82.1	1,054
11a	2.64	2.08	10.2	8.9	8.7	21.3	87.2	185
11b	2.64	2.08	10.2	9.4	8.7	21.3	92.2	159
8	2.60	2.28	5.3	4.3	4.5	12.1	81.2	573
8a	2.60	2.25	5.9	5.3	5.0	13.3	89.8	390
8b	2.60	2.22	6.5	5.9	5.5	14.5	90.6	111

[a] (Basic data from Tables 17 and 18)
[b] Estimated future maximum saturation 90% of water capacity. Estimated future maximum saturation of the pavement base 85% of water capacity.

Note.—Numbers marked "a" and "b" are from the same samples, but with different pore space and saturation.

apparent density is constant. This being true, the pore space for a given bulk density must also be constant.

Table 22 is compiled from a graph said to show that a specimen of earth may, under certain conditions, be compacted too much for producing the desired strength. The outstanding properties of Table 22 are summarized as follows:

1. The strength of a specimen of earth containing a given percentage of water invariably decreases with increasing pore space.

TABLE 22.—EFFECT OF PORE SPACE AND SATURATION ON THE STRENGTH OF ROADBEDS

No.	Density, in grams per cubic centimeter		Weight percent water			Pore space		Strength CBR, %
	Apparent	Bulk	Capacity	Content (molding)	Content (future)[a]	Total %	Saturation %	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	2.72	1.84	17.6	13	15.8	32.4	73.8	80 U[b]
2		1.84	17.6	14	15.8	32.4	79.5	70 U
3		1.84	17.6	15	15.8	32.4	85.1	52 U
4	Group 1	1.84	17.6	16	15.8	32.4	90.9	20 U
5		1.84	17.6	14		32.4		26 S[c]
6		1.84	17.6	15		32.4		33 S
7		1.84	17.6	16		32.4		13 S
8	2.72	1.76	20.0	12	18.0	35.2	60.0	62 U
9		1.76	20.0	13	18.0	35.2	65.0	50 U
10		1.76	20.0	14	18.0	35.2	70.0	43 U
11		1.76	20.0	15	18.0	35.2	75.0	36 U
12		1.76	20.0	16	18.0	35.2	80.0	29 U
13	Group 2	1.76	20.0	17	18.0	35.2	85.0	13 U
14		1.76	20.0	12		35.2		4 S
15		1.76	20.0	13		35.2		5 S
16		1.76	20.0	14		35.2		7 S
17		1.76	20.0	15		35.2		9 S
18		1.76	20.0	16		35.2		15 S
19		1.76	20.0	17		35.2		10 S
20	2.72	1.68	22.8	12	20.5	38.3	52.6	40 U
21		1.68	22.8	13	20.5	38.3	57.0	34 U
22		1.68	22.8	14	20.5	38.3	61.4	30 U
23		1.68	22.8	15	20.5	38.3	65.7	28 U
24		1.68	22.8	16	20.5	38.3	70.1	24 U
25		1.68	22.8	17	20.5	38.3	74.5	18 U
26	Group 3	1.68	22.8	20	20.5	38.3	87.6	3 U
27		1.68	22.8	12		38.3		2.5S
28		1.68	22.8	13		38.3		3.5S
29		1.68	22.8	14		38.3		4 S
30		1.68	22.8	15		38.3		5 S
31		1.68	22.8	16		38.3		6.5S
32		1.68	22.8	20		38.3		4 S
33	2.72	1.60	25.8	12	23.2	41.2	46.5	28 U
34		1.60	25.8	13	23.2	41.2	50.4	24 U
35		1.60	25.8	14	23.2	41.2	54.2	22 U
36		1.60	25.8	15	23.2	41.2	58.1	20 U
37		1.60	25.8	16	23.2	41.2	62.0	18 U
38		1.60	25.8	18	23.2	41.2	69.8	11 U
39	Group 4	1.60	25.8	20	23.2	41.2	77.5	8 U
40		1.60	25.8	12		41.2		2 S
41		1.60	25.8	13		41.2		2.5S
42		1.60	25.8	14		41.2		3 S
43		1.60	25.8	15		41.2		3.5S
44		1.60	25.8	16		41.2		4 S
45		1.60	25.8	20		41.2		5 S

[a] Future maximum water content estimated at 90% of water capacity.

[b] U = unsoaked strength. [c] S = strength after soaking. Basic data from "Stabilization of Materials by Compaction," by Willard J. Turnbull and Charles R. Foster, Transactions, ASCE, Vol. 123, 1958, Fig. 2. Material : Weathered Loess near Vicksburg, Miss.

2. The strength of a specimen of earth containing a given percentage of pore space invariably decreases with increasing saturation.

3. The strength of a specimen of earth is the greatest during wet or dry weather when its pore space is the minimum and its saturation has reduced pore suction to zero.

4. For most earthy matter, zero pore suction may well be regarded as approximately 90% saturation.

5. The least ability of a specimen of earth with a given pore space to resist loads during wet weather is when its pore space is entirely filled with dry air at the beginning of wet weather.

6. The greatest potential strength to be expected of any specimen in the table during wet weather is estimated to be one with values lying between Nos. 3 and 4. Such a specimen would have a pore space of about 32%, a saturation value of about 88% and a CBR strength of about 36%. During wet weather, the table indicates a CBR strength of about 23%.

7. The least potential strength to be expected of any specimen in the table is No. 33 with a pore space of 41.2%, a saturation of 46.5% and a CBR strength of 28%. During wet weather its soaked counterpart, No. 40, is shown by the table to have a CBR strength of 2%.

8. Although No. 4 with a molding water content of 16% is shown to be 90.9% saturated, its counterpart, No. 7, is shown to have a CBR strength of only 13% after soaking. The loss in strength from 20% to 13% seems too great. As the exact conditions of testing are unknown, it would seem that the soaked specimen may have lost water before soaking, a condition that could have produced a higher saturation and, therefore, an additional loss in strength. Also it has been shown (38) that specimens during the period of soaking do not absorb equal percentages of water near the surface and in the interior. This also could cause discrepancies in strength. It would, therefore, seem that one or both of the causes might be responsible for the discrepancy.

9. In contrast, Nos. 13 and 19 have nearly identical values, a condition similar to that in a number of cases.

10. Concerning the great loss in strength of the soaked counterpart of specimen No. 33, summarized under G, the following comment is offered: as group 4 has the highest percentage of pore space of any other group and lowest molding or starting-out-content of water—the lowest saturation percent—it seems logical that it should be expected to absorb the greatest volume of water; therefore, it should be expected to lose more strength on soaking than any other specimen.

11. The higher the percentage of pore space and the lower the percentage of saturation of a specimen of earth at the time it comes in contact with water, the greater its consequent percentage of absorption, swell and loss of strength.

12. The lower the percentage of pore space and the higher the percentage of saturation of a specimen of earth at the time it comes in contact with water, the less its consequent percentage of absorption, swell, and loss in strength.

Airport Runways Withstand a Flood.—In the year 1943, the runways of the Omaha Airport, Omaha, Nebraska became submerged from 3 ft to 7 ft during a period of 10 days to 17 days. The roadbed of the runways consists of a 12 in. soil compacted subbase with a PI of 18% to 29% laid on a Missouri River silty clay; a 6 in. compacted sand-clay-gravel base, and a 2 in. to 3 in. bituminous surface.

From test data obtained by Campen and his associates immediately after the subsidence of the flood, characteristic conditions are shown in Table 23 and a summary in Table 24. This research is fortunate in having access to test data on runways after they have been subjected to the flood conditions previously described. Attention is called to the limited pore space and its saturation by the subbase and the base also. In reviewing these values, one

TABLE 23.—RELATIONSHIP OF PORE SPACE AND SATURATION OF OMAHA RUNWAYS[a,b]

Density, in grams per cubic centimeter		Weight percent water		Pore space	
Apparent (1)	Bulk (2)	Capacity (3)	Content (4)	Total percentage (5)	Saturation percentage (6)
Subbase					
2.62	1.85+	15.9-	14.4	29.4-	90.5
2.62	1.78	18.0	11.8-	32.0	98.7
2.62	1.75	19.0	16.2	33.2	85.2-
2.62	1.72	20.0	19.7	34.4	98.7
2.62	1.69	21.0	20.9	35.5	99.5+
2.62	1.69	21.0	19.6	35.5	93.2
2.62	1.69	21.0	20.2	35.5	96.1
2.62	1.68	21.3	20.5	35.8	96.2
2.62	1.68	21.3	21.0	35.8	98.5
2.62	1.66-	22.0+	21.5+	36.5+	97.6
Avg.	1.72	20.0	19.6	34.4	95.4
Subbase					
2.63	1.85+	16.0-	15.0	29.6-	93.6
2.63	1.83	16.6	14.4-	30.4	86.6-
2.63	1.82	16.9	16.0	30.8	94.6
2.63	1.81	17.2	16.2	31.1	94.1
2.63	1.80	17.5	17.4	31.5	99.5+
2.63	1.78	18.1	17.3	32.3	95.5
2.63	1.78	18.1	17.6	32.3	97.2
2.63	1.78	18.1	17.5	32.3	96.6
2.63	1.78	18.1	17.5	32.3	96.6
2.63	1.77	18.4	17.8	32.6	96.7
2.63	1.71-	20.5+	19.7+	35.0+	96.0
Avg.	1.74	17.8	16.9	31.8	95.2

[a] Maximum, ; minimum, -. Basic data from "The Behavior of Densified Soil-Water Mixtures Under Very Adverse Conditions," by W. H. Campen and J. R. Smith, Proceedings, Highway Research Bd., Vol. 23, p. 450. Note. Columns 3, 5, and 6 computed for this paper by Ira B. Mullis.
[b] After Flood of 1943

should remember that though pore suction aided in the absorption of water, the hydraulic head must have contributed most. Another important feature of Table 24 is that shown in column 5 that indicates that a grand average of only 92.6% of the pore space became filled, even under the very adverse conditions imposed by the flood. Attention is called also to the close approach of estimated and actual values shown in columns 2 and 3. Of the bases, similar com-

ments seem applicable, though it is apparent that as usual the excellent bases seem to offer more resistance to the penetration of water than do the clays. Only one or two specimens showed a saturation of 100%.

States and Properties of Clay.—Nature has decreed that solid rocks can be decomposed into clay mostly by the processes of drying and wetting and by heating and cooling. Therefore, rocks hidden deep in the earth where change is the slightest, retain their strength the longest and those exposed to rapid change at the surface are most rapidly changed to clay-like matter. As clays, unlike rocks, have been weathered to the end-point, they undergo little chemical change; however, under a favorable environment they undergo enormous physical change. With varying amounts of water, clay may range from a slurry to a solid with little water. As a slurry, it can sustain loads only by its buoyant force; but when its pore space is reduced to the minimum, and almost filled with water, its toughness and strength are at the maximum. In the powdered state, it is extremely weak; but when the pore space is small and filled with air, it is a strong brittle solid.

TABLE 24.—AVERAGE PORE SPACE, WATER CONTENT AND SATURATION OF RUNWAYS, OMAHA AIRPORT AFTER MISSOURI RIVER FLOOD OF 1943.

	Weight percent water			Pore space	
	Capacity (1)	Content (actual) (2)	Content (future)[a] (3)	Total percentage (4)	Saturation percentage (5)
Subbase—No. tests, 75					
Maximum,	23.4	22.3	21.8	37.6	99.0
Minimum,	14.6	13.2	13.6	27.6	75.6
Average,	19.3	17.7	17.7	33.7	92.6
Base—No. tests, 17					
Maximum,	8.5	7.0	7.6	18.2	95.9
Minimum,	5.8	4.9	5.0	13.1	79.4
Average,	7.1	5.8	5.6	15.0	87.3

[a] The estimated maximum future water content to be expected is 90% of the water capacity of the pore spaces. Values in the table are from "The Behavior of Densified Soil-Water Mixtures Under Very Adverse Conditions," by W. H. Campen and J. R. Smith, Proceedings, Highway Research Board, Natl. Research Council, Vol. 23, 1943, p. 450.

Like solid rock, a particle of clay can swell or shrink very little; but when it contains a fluid such as air or water, it shrinks or swells in proportion to the loss or gain of fluid content. Consequently, no change in volume can occur if the fluid content remains constant regardless of the volume of pore space. However, if the body of clay is to be made and kept strong, pore space must be correspondingly small and its percentage of saturation kept constant.

Table 25 shows that when pore space and saturation are high and drying occurs, shrinkage is greatest. Due to variation to be expected in all tests, some variation is shown, but the trend can hardly be questioned.

Fig. 5 shows a loose gumbo that absorbs large quantities of water and undergoes considerable volume change during wetting and drying. When puddled, shrinkage usually occurs in the form of fissures. Wherever gumbo clay is found, its strength varies enormously with water content, but this variation

TABLE 25.—EFFECT OF PERCENT OF PORE SPACE AND SATURATION ON SHRINKAGE

Density, in grams per cubic centimeter		Weight percent water			Pore space		Volumetric percent shrinkage
Apparent	Bulk	Capacity	Content (original)	Content (future)[a]	Total percentage	Saturation percentage	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
2.65[b]	1.70	21.0	10	18.9	35.8	47.6	5.7
2.65	1.70	21.0	12	18.9	35.8	57.6	4.4
2.65	1.70	21.0	14	18.9	35.8	66.6	2.6
2.65	1.70	21.0	16	18.9	35.8	76.1	1.6
2.65	1.70	21.0	18	18.9	35.8	85.5	0.7
2.65[b]	1.80	17.8	10	16.0	32.0	56.1	5.6
2.65	1.80	17.8	12	16.0	32.0	67.4	4.1
2.65	1.80	17.8	14	16.0	32.0	78.5	2.5
2.65	1.80	17.8	16	16.0	32.0	89.7	1.4
2.65	1.80	17.8	18	16.0	32.0	99.0	0.0
2.65[b]	1.85	16.3	10	14.7	30.0	61.3	5.5
2.65	1.85	16.3	12	14.7	30.0	73.6	3.6
2.65	1.85	16.3	14	14.7	30.0	85.7	1.5
2.65	1.85	16.3	16	14.7	30.0	98.1	0.8
2.65	1.85	16.3	18	14.7	30.0	?	0.0
2.65[b]	1.60	24.6	10	22.2	39.5	40.6	5.5
2.65	1.60	24.6	12	22.2	39.5	48.7	3.6
2.65	1.60	24.6	14	22.2	39.5	56.9	2.6
2.65	1.60	24.6	16	22.2	39.5	65.0	1.8
2.65	1.60	24.6	18	22.2	39.5	73.1	1.2
2.65[b]	1.51	28.4	10	25.6	43.0	35.2	1.0[c]
2.65	1.51	28.4	12	25.6	43.0	42.2	3.2
2.65	1.51	28.4	14	25.6	43.0	49.2	2.6
2.65	1.51	28.4	16	25.6	43.0	56.3	1.6
2.65	1.51	28.4	18	25.6	43.0	63.4	---

[a] Estimated future maximum saturation 90% of water capacity.
[b] Assumed value. [c] Value believed to be the lowest in the group due to large pore space and low saturation.

Note.—Bulk density, percent moisture when compacted and percent of shrinkage from "The Results of Tests to Determine the Expansive Properties of Soils," by Harold Allen and A. W. Johnson, Proceedings, Highway Research Bd., Natl. Research Council, 1936, p. 220. Numerical values in the above table taken from the discussion by Delbert L. Lacey of "Design of Roadbeds," by Ira B. Mullis, Transactions, ASCE, Vol. 111, 1946, Table 8.

in strength is often controlled by the use of consolidated sand-clay mixtures to such an extent that these roadbeds carry great loads. Fig. 6 shows a sand-clay road surface on a gumbo roadbed characterized by the lack of loose material. Due to the strong bond of gumbo to the sand or gravel, these surfaces resist dry weather better than do less adhesive clays.

FIG. 5.—A POROUS GUMBO CLAY THAT ABSORBS MUCH WATER

FIG. 6.—A CONSOLIDATED SAND-CLAY SURFACE ON A GUMBO ROADBED

Does Capillary Water Accumulate Under Pavements?—Additional support is given to premises 17, 18, and 19 by Oscar Edward Meinzer (40) when he states that if the capillary fringe is near enough to the ground surface to lose water by evaporation or by the absorption of roots of plants, there is a continuous movement of water from the water table upward through the capillary interstices.

The Encyclopedia Britannica (41), says the cohesion theory of the assent of sap requires only that one membrane is evaporating into dry air, another membrane is in contact with a water supply, and the two connected by a continuous water thread under such conditions that it can support a tensile stress.

FIG. 7.—A HORSE ENGULFED IN A QUAGMIRE AT THE FOOT OF A SNOW-COVERED SLOPE

In view of the experimental evidence that evaporation is an essential to the movement of capillary water and with no experimental evidence to the contrary, it seems that capillary action cannot produce an accumulation of water at the discharge end of a wick-like material of any kind including earth.

Frost Action in Roadbeds.—Under the side heading, "Frost Action," frost action is discussed largely on the basis of experience gained during this research, but it will now be confined to the research work of others.

Frost Effects on Solid and Unconsolidated Bodies.—Earnest de K. Leffingwell (42), after several years of exploration and research in northern Alaska, notes that, of course, ground ice is not to be expected in hard rock, but in

what would be considered as unconsolidated deposits, if they had not already been solidified by ice. Of the unconsolidated deposits, some were more favorable to frost action than others. Coarse sand and gravel deposits seem to have much less ice than clay and silt. The most favorable material is muck.

Small Pores and Cracks Least Resistant to Frost.—Stephen Taber (43), another research worker on frost action in Alaska, states that ice crystals can best develop in relatively permeable, fine-grain rocks having numerous small pore spaces. Normally, fine-grain rocks that are highly impermeable, and coarse-texture rocks with relatively large open spaces, are most resistant to frost action. Due to lack of water, frost effects are practically absent on bare rocky ridges and cliffs; but at the foot of steep slopes where water accumulates, frost destruction is relatively rapid. The ice content of silts may be as much as 80% by volume and the average is probably in excess of 50%. Such silts on thawing turn to semi-liquid mud that flows where opportunity exists for escape.

In support of the statement by Taber concerning the relatively rapid destruction at the foot of slopes, Fig. 7 is shown. The horse engulfed in the quagmire located at the foot of a slope illustrates the result of frequent

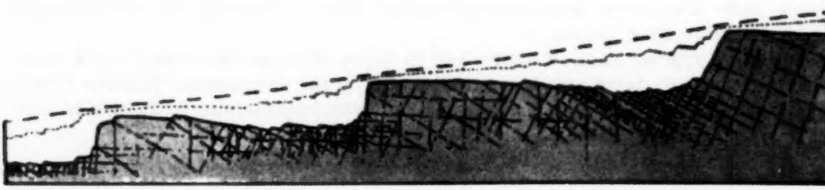

FIG. 8.—EFFECT OF ROCK JOINTS ON GLACIAL QUARRYING

freezes and thaws. The dark belt near the top of the photograph shows ground on which the snow has completely thawed. Below the belt, snow is available for additional thaw water. South-facing slopes, because of their more frequent exposures to the sun usually supply more thaw water during alternate freezes than do north-facing slopes that remain frozen until evaporation lessens the runoff. Of course, many repetitions of freezes and thaws, enlarge pore space and water refills it until the area becomes a treacherous quagmire. Where cattle have access to these quagmires they oftentimes become engulfed.

Glacial Quarrying in Jointed Rocks.—According to Richard F. Flint (44) well-consolidated clay-rich till, virtually free of joints resists quarrying almost perfectly and, where glacially eroded, shows the effects of abrasion only. Once the glacier has removed the weathered mantle and has worked down through the open-jointed bedrock beneath, the quarrying becomes sharply reduced and continues to diminish as joints become tighter with depth. Fig. 8, after F. E. Matthes, shows how frost-welding, quarrying or plucking-out in the more porous or jointed rocks are changed into step-like forms during glaciation on slopes. The steps are shown to be controlled by the unequal distribution of joints and other planes of weakness that cut the bedrock, though other factors may affect the production of such steps. The dashed line indi-

cates the valley profile before glaciation; the dotted line indicates the profile during an early phase of the glaciation.

Although rocks of limited pore space do resist frost action to a remarkable degree, conditions occur that cause serious rupture. As rocks at considerable depths are likely to be highly saturated with water, many quarries supplying dimension stone in very cold regions are forced to discontinue quarrying to avoid destructive frost rupture before adequate evaporation can occur.

Following this analogy, a consolidated body of clay with pore space more than 90% saturated must also rupture under a temperature that freezes the entire water content. However, if a roadbed is carefully constructed, experience teaches that 100% saturation is not likely to attend construction. However, when roadbed construction is carelessly done so that pore spaces contain large quantities of air, vibration from traffic, changes in barometric pressure and precipitation are likely to produce a saturation that is sufficient to cause considerable damage, especially during a deep freeze immediately following considerable precipitation. On the other hand, when pore space is reduced to small percentages and saturation is ample, additional saturation seems to be improbable as is indicated in Table 22.

Conclusions.—As the bedrock foundation of the paper rests on premises 1 through 21 near the beginning and in the section entitled "Reduction of Pore Space Near Saturation Increases Strength," items 1 through 12, are regarded as appropriate conclusions.

Grateful appreciation is expressed to those who have provided much basic data and also to those who have contributed to the discussion. Without these, the paper, in its present form, would have been impossible. Gratitude is also expressed to L. Polk Denmark for his preparation of Fig. 4.

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DEVELOPING ALTERNATIVE URBAN TRANSPORTATION SYSTEMS[a]

Closure by William R. McCononchie

WILLIAM R. McCONONCHIE,[6] F. ASCE.—Mr. Cherniack has made a very convincing presentation of the case for express buses on freeways and parkways as a permanent plan for providing rapid transit in metropolitan areas. This type of operation was carefully considered and recommended for several of the lighter routes in the Washington, D. C. study. The writer believes, however, that the findings relative to use of buses on the heavier routes were not only valid for Washington, but also for other cities of comparable size. For the four heaviest corridors in Washington, rail rapid transit was recommended rather than express buses for the following reasons:

1. The freeways on which center mall rail rapid transit was recommended would be so heavily used by automobiles and trucks soon after completion that introduction of 200 buses per hour to 250 buses per hour in each direction during rush hours would make intolerable a situation that will be difficult at best.
2. Maintenance of satisfactory speed and regularity of bus service on a badly overloaded freeway is virtually impossible. People then turn to the use of their own automobiles to gain the flexibility needed to choose less congested routes, and bus patronage quickly deteriorates to the point at which no bus service is needed.
3. Express buses arriving at the downtown end of the route must either be absorbed in the general traffic stream on the surface streets, that is not an inviting prospect under today's worsening traffic conditions, or they must have a terminal such as that operated in New York City by the Port of New York Authority with scores of bus berths as well as a subway system to distribute the passengers. In Washington, it was estimated that local buses, plus the express buses from the routes recommended for this type of operation would use all of the available space on the surface streets by 1980. It was concluded, therefore, that compounding downtown congestion with additional buses would be unwise.
4. Once a freeway is opened to traffic it is incumbent on responsible officials to keep it in operation under any and all circumstances. This policy must be even more rigidly adhered to in connection with a public transportation facility because its patrons have fewer alternatives and the operating company has little flexibility. There might be insurmountable problems, therefore, in temporarily transferring an express bus line from the center mall of an expressway to roadways carrying general traffic while the center mall was being converted for rail rapid transit operation.

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a March 1960, by William R. McCononchie (Proc. Paper 2417).

6 Vice Pres., De Leuw, Cather and Co., Chicago, Ill.

The discussion accorded this paper is sincerely appreciated. In the opinion of the author, the points raised by Hoover, Forsythe, and Cherniack are not so much indicative of differences of opinion on what is obviously a very lively topic, but rather they are vivid illustrations of the careful and exhaustive studies that must follow the preliminary planning of a comprehensive transportation system. As the project is carried to advanced stages of planning, each step will have to be considered in the light of recent developments and experience in this fast-moving field.

LTS DESIGN OF CONTINUALLY REINFORCED CONCRETE PAVEMENT[a]

Discussion by William Zuk, Howard Newlon, Jr.,
R. A. Mitchell, and Bengt F. Friberg

WILLIAM ZUK,[4] M. ASCE, HOWARD NEWLON, JR.,[5] A.M. ASCE, and R. A. MITCHELL,[6] A.M. ASCE.—The authors have presented a worthy attempt to place the design of continuously reinforced concrete pavements on a rational basis. As all who have worked with this complex subject are aware, many of the areas considered by the authors have been the basis of speculation for several years and it is particularly valuable to have available data based on pavements actually constructed.

It is most gratifying to see the results (Fig. 9) showing the important role that bond area plays in reducing crack spacing. In past studies of the behavior of continuous pavements, bond has really never been given the emphasis it deserves. By increasing bond area, without changing the cross sectional area of the steel, one can greatly increase the general efficiency of the pavement by decreasing the crack width.

From their experience the writers are fully aware of the extreme complexity of the problem faced by the authors in developing their design method and so offer the following observations not in the spirit of criticism but rather with the hope that a thorough discussion will aid in a more rapid evolution of a sound and generally accepted design procedure.

Regarding the basis of the design approach presented by the authors, issue is taken with the fundamental concept of attempting to apply Westergaard's equations for plain concrete pavements to such a radically different type of pavement as a continuously reinforced one. The features of closely spaced cracks and high percentages of reinforcing steel cause the continuous pavement to behave in a manner fundamentally different from that of a plain pavement. One of the writers' theories for an entirely new method of analysis (26) is perhaps extreme as it assumes ideal conditions of transverse cracks spaced at close intervals of approximately 1 ft (as compared to average crack spacings of about 5 ft), but it does signal the need for an up-to-date analysis of facts as they are. The rationale of modifying Westergaard's equations to account for reinforcement by merely subtracting the transformed area of steel (Eq. 10) is difficult to see.

From performance records of existing continuous pavements, the practical limits of pavement thickness and steel reinforcement are confined to a rather

a December 1960, by Benjamin F. McCullough and William B. Ledbetter (Proc. Paper 2677).

4 Prof. of Civ. Engrg., Univ. of Virginia and Highway Research Engr., Va. Council of Highway Investigation and Research, Charlottesville, Va.

5 Highway Research Engr., Va. Council of Highway Investigation and Research, Charlottesville, Va.

6 Highway Research Engr., Va. Council of Highway Investigation and Research, Charlottesville, Va.

narrow spectrum (thickness varying an inch or two about a mean of 7 in. or 8 in., and steel varying a few tenths of a per cent about a mean of approximately 0.7%). Thus, reliable design formulas must be quite precise in order to accrue the economic benefits of scientific analysis. If the formulas are rationally or practically unsound, the required precision is lost, and it may be wiser to simply place reliance on experience and judgment until such formulas are developed.

Still another point of question regards summary conclusion no. 6, concerning the location of the steel at the mid-depth of the slab. The various reasons presented by the authors for locating the steel at this position appear quite reasonable. However, other experimental evidence seems to indicate that such factors as temperature, shrinkage, and moisture swelling vary more near the top of the slab than they do near the bottom; that results in wider crack widths at the top than at the bottom. Thus, there is some thought that reinforcing placed a little above the mid-depth may tie the cracks somewhat more uniformly.

In their conclusion no. 7, as well as in no. 3 and no. 4, the authors indicate the desirability of specifying a maximum concrete strength and suggest a seven day value of 675 psi as measured by a modulus of rupture computed from center point loading. While this might theoretically be the best solution when all of the possibilities are considered, in certain respects it creates substantial problems of control and, more important, it ignores the fact that paving concrete must perform two functions; develop the proper strength and provide adequate durability. The first function is usually considered by designers, but the latter one is often more difficult to obtain in practice, particularly in locations in which repeated cycles of freezing and thawing occur, deicing chemicals are used, or the pavement is abraded by tire chains.

While the authors state in conclusion no. 4 that the specified values would depend on local conditions, the establishing of a definite maximum strength might be made at the expense of durability that the writers are sure was not the intent of the authors. For this reason certain points are set forth below to illustrate the caution necessary in establishing such a strength criterion.

In the first place, it is quite probable that control of concrete flexural strength within the limit of 100 psi as measured by the center point modulus of rupture is a desired end seldom achieved. The variability of flexure tests, as well as the normal variation of field concrete, has been widely discussed in the literature; it was well known to the authors as shown by their statement that "by calling for a 650 psi concrete, the contractor is forced to mix initially appproximately an 800 psi concrete and work down to insure compliance with the specifications." It is the reverse effect of this variation that is of concern to the writers. Committee 214 of the American Concrete Institute (ACI) has established that for good control an overall coefficient of variation of as much as 15% might be expected.[7] This value is based on compressive tests, and, for flexure tests, application of it is probably conservative. If the intent of the authors' specifications is that no tests should exceed 675 psi, this would mean that the average strength would be 465 psi and the minimum, 255 psi. If, on the other hand, it would be acceptable for approximately 16% of the tests to exceed the maximum specified strength, the average would then be 587 psi, but approximately 16% would still fall below 500 psi.

7 "Recommended Practice for Evaluation of Compression Test Results of Field Concrete," ACI Committee 214 Journal, ACI, Vol. 29, No. 1, July, 1957.

Aside from the control problems created by these variations, the durability of the lower strength concrete would require considerable study. This is true because durability is indirectly related to strength in the sense that, other factors being equal, the stronger the concrete, the more durable it is. Thus, any action that would decrease the strength of the concrete would also decrease the durability. The "other factors" cannot be ignored because two concretes of equal strength may not be equally durable and, in addition, different concrete materials when combined in the same proportions do not have the same potential strength.

As an example, the paving mix used in the state of Virginia is fairly typical of paving concrete in that it requires 6 sacks of cement per cubic yard, 5 gal of water per sack, and has a maximum size aggregate of 2 1/2 in. For this mix, average 7 day moduli of rupture for some materials have ranged as high as 900 psi without entrained air. It can be seen that in using the specification presented by the authors with these materials, it would be necessary either to reduce the cement content or to increase the water content of the mix, each of which would have an adverse effect on durability.

It is true that the use of air entrained concrete has greatly improved concrete durability and that many concretes of the recommended strength might be durable with the proper amount of entrained air, but even with air, the durability of the concrete with the relatively low strengths mentioned previously would be questionable.

One very important feature of design that is not discussed by the authors is the problem resulting from the relatively large end movements associated with extremely long sections of continuous pavement. These movements have been widely discussed in the literature (7),(21),(22),(23),(24),(26). One of the writers suggested, in 1958 (26), that subgrade anchors be used to control longitudinal slab movements in the vicinity of the slab end. The same year a 303 ft long continuously reinforced concrete sidewalk, incorporating five 18 in. deep anchors at each end, was built at the University of Virginia,[8] Charlottesville, Va. This continuous pavement is in excellent condition (as of May 1961) and a recent inspection indicated no measurable end movement. A theoretical and experimental study of the problem of control of pavement end movement is in progress (1961) at the Virginia Council of Highway Investigation and Research, and it is hoped that this study will lead to a rational analysis of pavement end anchors.

The writers are aware that Texas was one of the first states to install subgrade anchors on full scale pavements to control the longitudinal movement of concrete pavement slabs. A discussion of the experience gained in the use of these anchors would be of considerable interest.

BENGT F. FRIBERG,[9] F. ASCE.—Continuously reinforced concrete pavements in exploratory highway installations have demonstrated that a concrete pavement with fairly closely spaced tied cracks can maintain structureal integrity for a number of years under primary rural highway traffic. It is to be expected that designers interested in that type of pavement should be concerned with procedures for its design. The approach to design for three separate influences, load (L), temperature (T), and shrinkage (S), is in principle logical, but in the proposed application questionable.

8 "A Model Study of Continuously Reinforced Concrete Pavements," by Stafford Thornton, Va. Council of Highway Investigation and Research, Charlottesville, Va., November, 1958.

9 Cons. Engr., St. Louis, Mo.

Rational application of a structural system can either be based on experience or tests that have established its limits of capability, or on predictions of behavior related to calculable stress developments that can be fairly judged to occur. Experience has indicated common pavement thicknesses are sufficient, and generally yield the crack spacings that seem to give lasting pavements, as well as, to a limited extent, the longer crack spacings that have caused trouble on poor subgrades. However, research goes on for optimum amounts and size of steel to promote the desired crack spacing. It is not known, (in April 1961) whether or not the steel has any appreciable value beyond that of causing cracks at fairly close spacing and tying the elements together so that no cracks predominate and become excessively wide. Nevertheless, experience provides for a rational, though not necessarily the most economical, application of continuous pavement construction.

On the other hand, design cannot be based on behavior and stress predictions unless the assumptions of conditions on which analytical procedures are based can be fairly considered to occur. Formulas cannot be justified merely by the fact of specific data yielding results falling within the range of experience; they must be doubted even more if they include extravagant assumptions to arrive at such answers. The lack of knowledge does not justify the use of known but inapplicable data.

The design procedures in the paper for continuously reinforced pavements are as follows:

1. Determination of pavement thickness by Westergaard formulas applicable to interior and edge loadings;
2. Modification of depth downwardly by an equivalent concrete area for the steel, used especially to modify the required edge thickness;
3. Determination of longitudinal steel to promote cracks at close spacing on simultaneous stresses equal to concrete tension strength and steel yield point strength.

The proposed design procedures are questioned for the following reasons:

1. Stresses in longitudinal direction for the intended close spacing of cracks cannot be realistically predicted by the Westergaard stress formulas. Load stresses might reach critical values in the transverse direction for very small crack spacing, but the formulas are useless for those stresses. The Westergaard formulas for interior and edge loading do apply for long crack spacings, which are not desirable; intermediate cracks, caused by intentional excessive loads, may in fact serve a useful design purpose on continuously reinforced pavements. Corner load conditions, however, appear to occur as an important and sometimes critical stress factor, that might be fairly well predicted by the Westergaard corner load stress formulas, with appropriate judgment with regard to load transfer across cracks.

2. The modification of concrete thicknesses by the equivalent area procedure, cited by the authors, relates only to steel and concrete areas away from and at equivalent distance from the neutral axis. Concrete in flexure at top or bottom cannot be displaced by steel that is at mid-depth near the neutral axis and offers no appreciable flexural resistance.

3. Steel requirements, to promote transverse cracking at close spacing, have been based on independently assumed stresses in the concrete and the steel, equal to the available strengths. The proposed procedure disregards

the interdependency of the steel stresses at cracks and the concrete stresses between cracks. Except for possible long crack spacing at early age, much greater than that intended in the type of construction, steel yield point stresses might never be reached at low temperature. The computed percentage of steel may be too low, so that average crack spacing remains greater than intended, if it is assumed that steel yield point stress can always be approached as a crack inducing tool.

It does not appear necessary to amplify these comments with respect to pavement depth determination. The fallacy in the proposed modification of depth would hardly require discussion. An investigation of actual flexural stresses in continuously reinforced concrete is beyond the scope of the discussion. Steel behavior for promotion of cracks and calculable effects of that steel on concrete movements are especially pertinent to the subject for discussion. Presentation in this discussion of more complete relationships than cited by the authors between concrete and steel direct stresses in a long cracked pavement restrained against over-all length changes, and the associated changes in crack widths, may be of value for appraisal of the construction and further development of continuous pavements.

Steel and Concrete Restraint Stresses and Crack Widths.—Determination of steel ratio to "control volume changes,"—more correctly—to induce closely spaced cracks,—has been based on studies by Vetter (3). Inasmuch as the Vetter studies have been cited in many publications on continuously reinforced pavements, a resume of the stresses and movements is given below, so that the complete relationships may be seen (the Vetter formulas do not give all derivations and equations), rather than the limited and, in part, incorrect applications included in the paper.

Stresses and movements relating to restrained temperature contraction alone, have been given by Friberg (28). For any distance between cracks L and bar bonding characteristics, that is defined by bond modulus "U," meaning change in steel stress for each inch of bar length in active bond, steel and concrete stresses and length of active bond are interrelated. Actual stresses can be determined by conventional methods, without the possibility for arbitrary stresses assumed by the authors.

The basic principle of the Vetter studies is that steel stress at crack is transferred to the concrete on each side by bond, by constant bond stress, with the length in active bond increasing with increasing stress at the cracks. This characteristic behavior has been confirmed by studies of bond tests reviewed by the writer (28). The bond stress, or bond modulus, is characteristic for bar size and surface, and concrete, and is not a conventional design bond stress. It could be subject to great decrease with repeated load application, that does not appear to have been investigated for the substantial stress variations in continuous reinforcement; however, such a possible decrease is indicated by bond failures observed in Pennsylvania.

Complete stress solutions and crack width data are given in Table 1 for shrinkage z, and temperature drop t, using conventional symbols indicated in the paper, separate coefficients for thermal expansion of the concrete e_c, and steel e_s, and additional stress symbols, and so on, given in Table 1.

For composite shrinkage unrestrained at cracks, elastic properties in the steel and the concrete determine the opposed concrete and steel stresses in the bonded portions; for any further stress changes in the bonded portions the

change in steel stress is related to the change in concrete stress as E_s/E_c, with further incremental stress due to difference in thermal coefficients. Considering this relation, and equilibria, all stresses, and movements (bond slips) near cracks can be computed. During shrinkage or temperature drop no compression can occur across cracks. For constant bond stress, the length of active bond on each side of the crack is then directly related to the concrete stress, due to transfer of stress from the steel across the cracks to the concrete by bond:

$$C = \frac{F_c}{p\,U} \qquad (17)$$

The total shortening of an element L between cracks, for $L \geq 2\,C$, or opening of the crack between two elements of length L, is

$$V = L\left(z + e_c\,t\right) - \frac{F_c}{E_c}\,(L - C) \qquad (18)$$

If F_c is known, both active bond length and crack openings can be computed directly with Eqs. 17 and 18.

TABLE 1

(1)	General (2)	Shrinkage, Unrestrained at Cracks (3)	Shrinkage, Restrained at Cracks (4)	Temperature Contraction Restrained at Cracks (5)	Combined Shrinkage and Temperature Restraint (6)
Stress in concrete between cracks	F_c	f_{zoc}	f_{zc}	f_{tc}	f_c
Stress in bonded steel between cracks	F_b	f_{zob}	f_{zb}	f_{tb}	f_b
Stress in steel at cracks	F_s	0	f_{zs}	f_{ts}	f_s
Length of active bond on each side of crack	C	C_o	C_z	C_t	C_r
Crack opening	V	V_o	V_z	V_t	V_r

The complete stress formulas for different suggested and applicable pavement length changes are as follows. For shrinkage without restraint at cracks:

$$f_{zoc} = p\,\frac{E_s\,z}{1 + n\,p} \qquad (19)$$

$$C_o = \frac{E_s\,z}{U\,(1 + n\,p)} \qquad (20)$$

$$f_{zob} = -\,\frac{E_s\,z}{1 + n\,p} \qquad (21)$$

and

$$V_o = L\ z\ \frac{1 + \frac{C_o}{L}\, n\, p}{1 + n\, p} \qquad \text{. . . . . (22)}$$

For shrinkage restrained by steel crossing cracks.

$$f_{zc} = \frac{E_c\ z\ n\ p\ L}{C_z + n\ p\ L} = p\ U\ L \left(\sqrt{\frac{E_s}{U\ L}\ z + \frac{(n\ p)^2}{(2)}} - \frac{n\ p}{2} \right) \qquad \text{. . . . (23)}$$

$$C_z = L \left(\sqrt{\frac{E_s}{U\ L}\ z + \frac{(n\ p)^2}{(2)}} - \frac{n\ p}{2} \right) \qquad \text{. . . . . (24)}$$

$$f_{zb} = - \frac{E_s\ z\ C_z}{C_z + n\ p\ L} \qquad \text{. . . . . (25)}$$

$$f_{zs} = \frac{E_s\ z\ (L - C_z)}{C_z + n\ p\ L} \qquad \text{. . . . . (26)}$$

and

$$V_z = L\ z\ \frac{C_z\ (1 + n\ p)}{C_z + n\ p\ L} \qquad \text{. . . . . (27)}$$

For temperature contraction restrained by steel crossing the crack

$$f_t = E_c\ e_c\ t\ \frac{n\ p\ L}{C_t + n\ p\ L} = p\ U\ L \left(\sqrt{\frac{E_s}{U\ L}\ e_c\ t + \frac{(n\ p)^2}{(2)}} - \frac{n\ p}{2} \right) \qquad \text{. . . (28)}$$

$$C_t = L \left(\sqrt{\frac{E_s}{U\ L}\ e_c\ t + \frac{(n\ p)^2}{(2)}} - \frac{n\ p}{2} \right) \qquad \text{. . . . . (29)}$$

$$f_{tb} = E_s\ e_s\ t \left(1 - \frac{e_c}{e_s}\ \frac{C_t}{C_t + n\ p\ L} \right) \qquad \text{. . . . . (30)}$$

$$f_{ts} = E_s\ e_s\ t \left(1 + \frac{e_c}{e_s}\ \frac{L - C_t}{C_t + n\ p\ L} \right) \qquad \text{. . . . . (31)}$$

and

$$V_t = L\ e_c\ t\ \frac{C_t\ (1 + n\ p)}{C_t + n\ p\ L} \qquad \text{. . . . . (32)}$$

For combined shrinkage and temperature contraction, restrained by steel crossing the cracks

$$f_c = E_c\ (z + e_c\ t)\ \frac{n\ p\ L}{C_r + n\ p\ L} \qquad \text{. . . . . (33)}$$

By setting $C_r = \frac{f_c}{p\ U}$ Eq. 33 yields

$$f_c = p\ U\ L \left(\sqrt{\frac{E_s}{U\ L}\ (z + e_c\ t) + \frac{(n\ p)^2}{(2)}} - \frac{n\ p}{2} \right) \qquad \text{. . . . . (34)}$$

$$C_r = L \left(\sqrt{\frac{E_s}{U\ L}\ (z + e_c\ t) + \frac{(n\ p)^2}{(2)}} - \frac{n\ p}{2} \right) \qquad \text{. . . . . (35)}$$

$$f_b = E_s\, e_s\, t - E_s \left(z + e_c\, t\right) \frac{C_r}{C_r + n\, p\, L} \quad \text{..........(36)}$$

$$f_s = E_s\, e_s\, t + E_s \left(z + e_c\, t\right) \frac{L - C_r}{C_r + n\, p\, L} \quad \text{.........(37)}$$

and

$$V_r = L\left(z + e_c\, t\right) \frac{C_r\,(1 + n\, p)}{C_r + n\, p\, L} = L\left(z + e_c\, t\right) \frac{1 + n\, p}{1 + n\, p\, \dfrac{L}{C_r}} \quad \text{....(38)}$$

Fig. 12 shows the steel and concrete stresses between cracks 80 in. apart for 0.0002 concrete shrinkage, and 50°F temperature decrease, for a fully

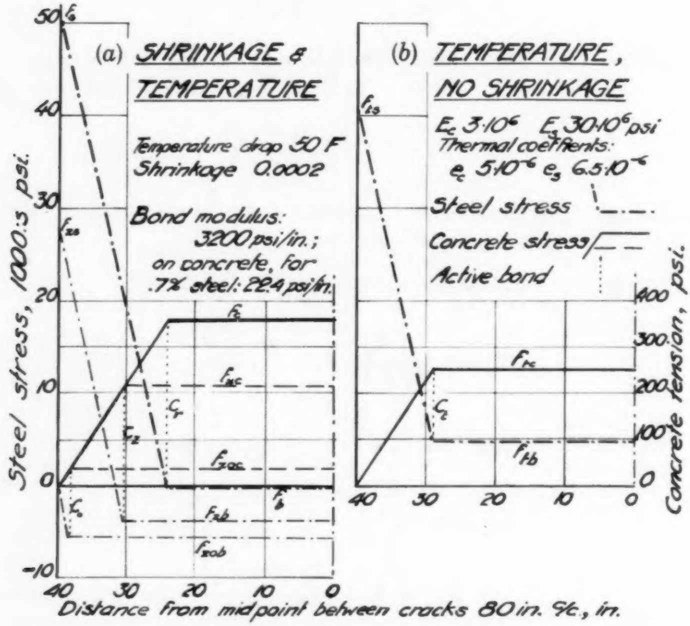

FIG. 12.—STEEL AND CONCRETE STRESSES IN FULLY RESTRAINED PAVEMENT, WITH CRACKS 80 IN. APART, AND 0.7% REINFORCEMENT OF 3,200 PSI PER IN. BOND MODULUS

restrained pavement with 0.7% steel, and assumed bond modulus 3200 psi per in. Computations give the value of maximum steel stress at crack as 50,000 psi, and concrete tension between cracks as 350 psi. The free shrinkage of 80 in. concrete would be 0.016 in., the restrained shrinkage for the reinforced concrete 0.011 in., and the crack opening for shrinkage and 50°F temperature decrease 0.028 in., compared to 0.036 in. without restraints. If there were no shrinkage ($z = 0$), the stresses computed previously for 50°F temperature decrease would be 250 psi concrete tension halfway between the

cracks, 40,800 psi steel tension at the cracks, and 4,800 psi tension in the bonded steel. (The steel's higher thermal coefficient accounts for 2,300 psi extra tension in the steel.)

According to Eq. 38, crack widths are not substantially smaller than would be computed for unrestrained movement, $L(z + e_c t)$, for crack spacings in the intended range. Accepting the Vetter analysis, it is incorrect to suggest that "cracks remain tightly closed," or that volume change can be "controlled" between the cracks. Fig. 7 is misleading because it does not show temperature above which the cracks were closed due to compression restraint. Continued measurements on the same project (22) show increasing crack widths, measured at high temperature, clear evidence that the cracks must have opened appreciably at low temperatures.

In the typical design, the authors obtain 0.68% longitudinal steel, using Eq. 5, that applies for cracking due to temperature change alone. Eq. 7 is obtained directly from Eqs. 33 and 17, solving for distance between cracks corresponding to the concrete tensile strength; however, the authors have neglected the fact that the equation is not valid if the concrete tensile strength is not reached between cracks. For 3/4-in. diameter bars, and 350 psi bond stress, assumed by the authors, the theoretical computed crack spacings (for a no. 6 bar it was 34.4 in.) are incorrect, because for that crack spacing the concrete tension stress half-way between the cracks could not exceed (p q u) (X/2), or 218 psi. The shortest crack spacing for which concrete tension strength could be reached for 0.68% steel and 350 psi bond on No. 6 bars would be 41 in., and would be equivalent to 50°F temperature decrease in addition to 0.0004 shrinkage, (Eq. 33), with a computed steel stress of 28,000 psi per Eq. 37. For further temperature decrease at any shorter crack spacing the cracks would continue to widen and the steel tension stresses would increase with decreasing temperature, but concrete tension stress would not increase in accordance with Vetter's analysis. Probable actual crack spacing (Fig. 8) is several-fold greater.

The concrete shrinkage, tension strength, and bond assumed by the authors are questionable. It is doubtful that shrinkage takes place in pavements to any great degree, or without associated change in applicable modulus of resistance. Observed highway pavement shrinkage in the Central states appears not to exceed 0.0001 and is noticeable only during the warm season. The swelling during the cold seasons approaches the same amount.

Equally questionable are the bond values. It is completely conjectural whether or not they are applicable to restraint stresses for repeated movements. The evidence (Pennsylvania) points to progressive bond slip spreading away from cracks, with greatly decreased restraints after the first cold season. If the properties assumed by the authors were representative, crack spacings would approximate the corrected theoretical values; however, as shown in Fig. 8, crack spacing in pavements with 0.70% steel should average about 80 in. at 3 yr to 5 yr age, (Curve B). Crack spacing averages about 6 ft at 10 yr (22) for 0.7% steel and concrete construction temperatures between 80°F and 90°F. The crack development is much closer to that indicated by temperature restraints alone, without shrinkage, and with supplemental minor cracking after the first winter due to traffic, with restraint stresses much decreased.

Continuous Pavement Indications.—Failures that have occurred on continuously reinforced pavements appear to be in two primary categories:

Structural break-up at locations of especially short crack spacing (Indiana, New Jersey, Pennsylvania), and corner and end fracturing at some "wide" cracks [at construction joints, or where extended bond failures have occurred (Illinois, Pennsylvania), or where cracks are far apart on poor subgrades (Illinois)]. In the interest of design economy, the determination of probable maximum crack width is a pertinent criterium for design.

With reference to Fig. 9, the authors state that the ratio of bond area to concrete volume Q should not be less than 0.03 for summer construction and 0.04 for late fall and early spring construction. Accepting the recommendation, and the data in Fig. 9 as representative, the optimum crack spacing would be about 7 ft for summer construction, and about 12 ft for winter construction. In either case a maximum average crack width of about 0.03 in. is indicated in cold weather. It could be inferred that maximum crack width of 0.03 in. has not been structurally detrimental, or permitted pumping to become serious, within available experience.

Observations of failures on continuously reinforced pavements give strong evidence that corner load stresses can become critical when there is insufficient load transfer at construction joints or at widely spaced cracks. Accordingly, there is ample reason to suggest the use of the Westergaard corner-load equations for determing pavement depth. The use of this formula allows for little or no load transfer if large crack spacing must be anticipated and at smooth construction joints, and allows for effective load transfer across cracks if crack spacing in the lower range can be expected before critical traffic periods.

ADDITIONAL BIBLIOGRAPHY

28. "Frictional Resistance under Concrete Pavements and Restraint Stresses in Long Reinforced Slabs," by Bengt F. Friberg, Proceedings, Highway Research Bd., Vol. 33, 1954.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

VOLUME 86 (1960)

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502(EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511(EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520(PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529(HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(SM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580(EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

SEPTEMBER: 2588(IR3), 2589(IR3), 2590(WW3), 2591(IR3), 2592(HW3), 2593(IR3), 2594(IR3), 2595(IR3), 2596(HW3), 2597(WW3), 2598(IR3), 2599(WW3), 2600(WW3), 2601(WW3), 2602(WW3), 2603(WW3), 2604(HW3), 2605(SA5), 2606(WW3), 2607(SA5), 2608(ST9), 2609(SA5)[c], 2610(IR3), 2611(WW3)[c], 2612(ST9)[c], 2613(IR3)[c], 2614(HW3)[c].

OCTOBER: 2615(EM5), 2616(EM5), 2617(ST10), 2618(SM5), 2619(EM5), 2620(EM5), 2621(ST10), 2622(EM5), 2623(SM5), 2624(EM5), 2625(SM5), 2626(SM5), 2627(EM5), 2628(EM5), 2629(ST10), 2630(ST10), 2631(PO5)[c], 2632(EM5)[c], 2633(ST10), 2634(ST10), 2635(ST10)[c], 2636(SM5)[c].

NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(WW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653(HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)[c], 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)[c], 2665(HY9)[c], 2666(SA6)[c], 2667(ST11)[c].

DECEMBER: 2668(ST12), 2669(IR4), 2670(SM6), 2671(IR4), 2672(IR4), 2673(IR4), 2674(ST12), 2675(EM6), 2676(IR4), 2677(HW4), 2678(ST12), 2679(EM6), 2680(ST12), 2681(SM6), 2682(IR4), 2683(SM6), 2684(SM6), 2685(IR4), 2686(EM6), 2687(EM6), 2688(EM6), 2689(EM6), 2690(EM6), 2691(EM6)[c], 2692(ST12), 2693(ST12), 2694(HW4)[c], 2695(IR4)[c], 2696(SM6)[c], 2697(ST12)[c].

VOLUME 87 (1961)

JANUARY: 2698(PP1), 2699(PP1), 2700(HY1), 2701(SA1), 2702(SU1), 2703(ST1), 2704(ST1), 2705(SU1), 2706(HY1), 2707(HY1), 2708(HY1), 2709(PO1), 2710(HY1), 2711(HY1), 2712(ST1), 2713(HY1), 2714(PO1), 2715(ST1), 2716(HY1), 2717(SA1), 2718(SA1), 2719(SU1)[c], 2720(SA1)[c], 2721(ST1), 2722(PP1)[c], 2733(PO1)[c], 2724(HY1)[c], 2725(ST1)[c].

FEBRUARY: 2726(WW1), 2727(EM1), 2728(EM1), 2729(WW1), 2730(WW1), 2731(EM1), 2732(SM1), 2733(WW1), 2734(SM1), 2735(EM1), 2736(EM1), 2737(PL1), 2738(PL1), 2739(PL1), 2740(PL1), 2741(EM1), 2742(ST2), 2743(EM1), 2744(WW1), 2745(WW1), 2746(SM1), 2747(WW1), 2748(EM1), 2749(WW1), 2750(WW1)[c], 2751(EM1)[c], 2752(SM1)[c], 2753(PL1)[c], 2754(ST2)[c], 2755(PL1).

MARCH: 2756(HY2), 2757(IR1), 2758(AT1), 2759(CO1), 2760(HY2), 2761(IR1), 2762(IR1), 2763(HY2), 2764(ST3), 2765(HY2), 2766(HW1), 2767(SA2), 2768(CO1), 2769(IR1), 2770(HY2), 2771(SA2), 2772(HY2), 2773(CO1), 2774(AT1), 2775(IR1), 2776(HY2), 2777(HY2), 2778(SA2), 2779(ST3), 2780(HY2), 2781(HY2)[c], 2782(HW1)[c], 2783(SA2)[c], 2784(CO1), 2785(CO1)[c], 2786(IR1)[c], 2787(ST3)[c], 2788(AT1)[c], 2789(HW1).

APRIL: 2790(EM2), 2791(SM2), 2792(SM2), 2793(SM2), 2794(SM2), 2795(SM2), 2796(SM2), 2797(SM2), 2798(EM2), 2799(EM2), 2800(EM2), 2801(EM2), 2802(ST4), 2803(EM2)[c], 2804(SM2)[c], 2805(ST4)[c].

MAY: 2806(SA3), 2807(WW2), 2808(HY3), 2809(WW2), 2810(HY3), 2811(WW2), 2812(HY3), 2813(WW2), 2814(HY3), 2815(WW2), 2816(HY3), 2817(HY3), 2818(SA3), 2819(WW2), 2820(SA3), 2821(WW2), 2822(WW2)[c], 2823(HY3), 2824(SA3), 2825(HY3), 2826(SA3)[c], 2827(HY3)[c].

JUNE: 2828(SM3), 2829(EM3), 2830(EM3), 2831(IR2), 2832(SM3), 2833(HW2), 2834(IR2), 2835(EM3), 2836(IR2), 2837(IR2), 2838(SM3), 2839(SM3)[c], 2840(IR2)[c], 2841(HW2)[c], 2842(EM3)[c], 2843(ST5), 2844(ST5), 2845(ST5), 2846(ST5)[c].

c. Discussion of several papers, grouped by divisions.